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Level 3 Assessment of Highway Girder Deck Bridges according to the Italian Guidelines: Influence of Transverse Load Distribution

Giuseppe Santarsiero *, Pierpaolo Albanese, Valentina Picciano, Giuseppe Ventura and Angelo Masi

Scuola di Ingegneria, Università degli Studi della Basilicata, viale dell'ateneo Lucano, 10, 85100 Potenza, Italy; pierpaoloalbanese@libero.it (P.A.); valentina.picciano@unibas.it (V.P.); giuseppe.ventura@unibas.it (G.V.); angelo.masi@unibas.it (A.M.)
* Correspondence: giuseppe.santarsiero@unibas.it

Abstract: The Italian Ministry of Infrastructure and Transportation adopted the guidelines on risk classification and management, safety assessment and monitoring of existing bridges through the Decree No. 578 dated 17 December 2020. This document must be used by all managing entities to prevent damage due to a lack of maintenance to these crucial components of the infrastructure system. The approach of the guidelines for existing bridges is developed across six levels, ranging from Level 0 to Level 5. The research work presented in this article is focused on Level 3, which pertains to preliminary assessments conducted on existing bridges. Through an automated procedure, the preliminary verification is performed by comparing bending and shear stress generated by traffic load schemes extracted from previous standards with the ones based on the current code. These loads are applied to a series of girder deck models, selected through a statistical study conducted on a database of bridges. Performance indices are derived from the comparison to evaluate the adequacy of previously designed and constructed structures by applying the load models specified in the current regulations for designing new bridges. The analysis results highlight a performance gap, which varies depending on the standard code at hand.

Keywords: existing bridge; reinforced concrete; girder deck; Italian Guidelines; transverse load distribution; bridge priorities; class of attention

Citation: Santarsiero, G.; Albanese, P.; Picciano, V.; Ventura, G.; Masi, A. Level 3 Assessment of Highway Girder Deck Bridges according to the Italian Guidelines: Influence of Transverse Load Distribution. *Buildings* **2023**, *13*, 1836. https://doi.org/10.3390/buildings13071836

Academic Editor: Oldrich Sucharda

Received: 15 June 2023
Revised: 8 July 2023
Accepted: 16 July 2023
Published: 20 July 2023

1. Introduction

Following the collapse of the Morandi Bridge in Italy, the issue of existing bridges' maintenance has been considered from the evaluation point of view at both a network level [1] and an individual one [2], with particular consideration given to material degradation. After this event, the Ministry of Infrastructure and Transportation enforced the guidelines for the classification and management of risk, safety assessment and maintenance of existing bridges through the Decree No. 578 (published on 17 December 2020) [3], which fully agrees with the Italian structural code [4]. These guidelines aim to prevent inadequate damage levels that can lead to catastrophic events through a procedure outlined in them. The multilevel approach adopted in the guidelines is an innovative aspect consisting of six levels, from zero to five, allowing for a progressive analysis of the structure.

Levels 0 and 1 provide information on the bridge by conducting accurate surveys and highlighting the presence of defects through visual inspections, as well as gathering all the available design documents. The information gathered in these levels is then used in Level 2 to evaluate the class of attention (CoA) of the bridge by combining four different risks (structure–foundation, seismic, flooding and landslides). It is worth noting that the class of attention evaluation considers the bridge's ageing and deterioration and, therefore, that the current state of an existing bridge is different from that specified in the design documentation. The class of attention assigned to bridges assumes the following possible results:

- Low
- Medium-Low
- Medium
- Medium-High
- High

Based on the assigned CoA, further measures must be undertaken. While periodic inspections are sufficient for low and medium-low CoA values, when the bridge has a medium or medium-high CoA, preliminary assessments are performed (Level 3). Based on the results of the preliminary evaluation, the managing entity decides whether a detailed evaluation of the bridge (Level 4) is necessary, which is mandatory in cases where the class of attention is high, providing recommendations for preventive measures and necessary interventions [5,6]. As known, a detailed assessment needs an in-depth knowledge of the structure derived from an in situ experimental testing campaign [7]. It is worth noting that in the case of high and medium-high CoA a permanent structural health monitoring system must be installed on the bridge [8,9]. Level 5 is devoted to the evaluations of the bridge at the road network level. Indeed, this level is not treated by the guidelines.

Various research studies have been devoted to the evaluation of bridges according to the guidelines from level 0 to level 2 [10]. For example, Scalbi et al. [11] analysed the impact of maintenance on the durability of a bridge. The objective of this study was to provide a method to streamline the evaluation procedure for assessing the condition of existing bridges and make it available to managing entities. Additionally, a research study by Fox et al. [12] critically analysed the guidelines to identify aspects that may require modification. This study was conducted during the experimental phase of the guidelines when they were applied to monitor the condition of existing bridges under the supervision of the High Council of Public Works. Another research work conducted by Abarca et al. [13] referred to the guidelines and defined a methodology for calculating losses due to the collapse of existing bridges. Finally, a research study by Renzi et al. [14] analysed the entire operational procedure described in the guidelines, with a particular focus on evaluating risk factors to determine the class of attention (CoA) of the structure, especially seismic risk.

Regarding Level 3, which involves the preliminary assessment of the bridge's capacity, less research is available. For example, a study by Buratti et al. [15] was based on a parametric analysis to demonstrate the variation in bending stress values considering both old standards and the traffic loads specified in the current regulation. Simplified initial assumptions were made in the study. Deterministic girder deck models were assumed for the parametric analysis, considering four different values of roadway width and four values of span length. The transverse load distribution procedure was not implemented and, consequently, the proportion of traffic loads transferred to each girder was not defined. This corresponds to the assumption that the bridge is made up of a single girder.

This article discusses the research study focused on preliminary assessments, as outlined in Level 3 of the guidelines. The methodology used is illustrated, which involves conducting a parametric study to compare the bending and shear stress values induced by traffic loads based on outdated design standards and the current one.

The girder deck models used in the parametric analysis are defined through statistical analysis and sampling, using data collected in a database constructed using samples from bridges along the highways in the Basilicata region (southern Italy). The study quantifies the performance gap between structures designed according to outdated standards and the application of traffic load models specified in the current one. Moreover, it shows that the capacity gap is significantly larger when considering the transverse load distribution [16]. The results may be useful to bridge managers for rapid Level 3 evaluations on bridges with geometric properties similar to those analysed in this study.

2. Database of Existing Bridges

This study focuses on bridges along the highways of the Basilicata region. Highways are also called "state roads", connecting the main towns of a region and across more than one region. They are very important to the Basilicata region, which has a scarce presence of motorways and trunk-type roads [10]. Along Basilicata highways, bridges are mostly multi-span simply supported girder deck type, similar to the viaduct shown in Figure 1.

Figure 1. View of a typical viaduct considered in the study.

Google Maps was used as a source of public information to identify the bridges along the main highways in Basilicata [17]. Through this tool, a search operation was conducted for the existing bridges. The total number of bridges identified at the end of the search was 163 structures. For each bridge or viaduct, it was possible to retrieve relevant information, including:

- Bridge name;
- Length of the structure;
- Number of spans;
- Deck width;
- Number of deck girders;
- Girder spacing;
- Structural material.

A further public source of information [18] is utilized to obtain the approximate construction year of each bridge. In fact, the bridge age has been assumed as that of the road as a first approximation, in order to establish the design code. The information collected in the database is shown in the following graphs.

Figure 2a shows the distribution of the average span length. Most of the bridges have span lengths between 20 and 50 m, while only a few of them have spans larger than 50 m. Thirty-six bridges have spans lower than 20 m, despite an average near 20 m. Therefore, in the following analyses, span length values between 20 and 50 m have been considered. This is in agreement with recent studies stating that girder deck-type bridges have an average span length of about 33 m [19].

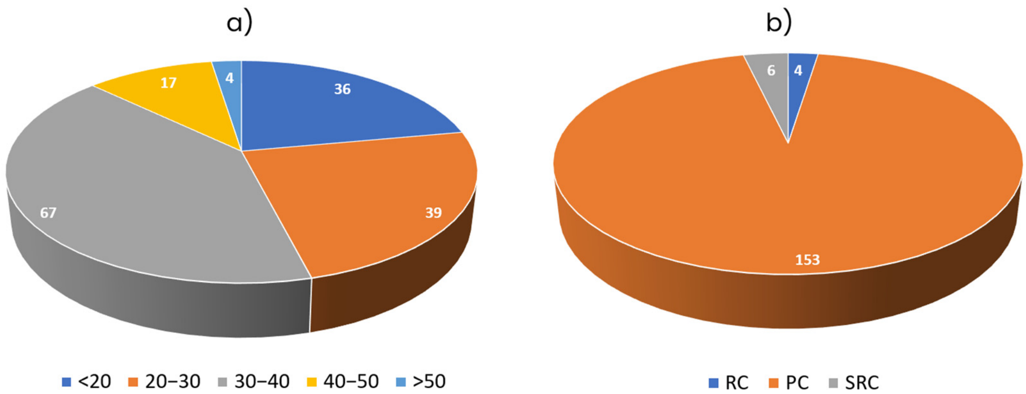


Figure 2. (**a**) Number of bridges by average span length (m) and (**b**) main deck material.

Figure 2b shows the material properties of the examined bridges. They are mostly made of prestressed concrete (PC) girders, while few units are in steel-reinforced concrete (SRC) or reinforced concrete (RC). All the bridges work on a simply supported static scheme. Data related to the type of piers is neglected since the following analyses are focused only on the deck.

Figure 3 shows the distribution of the number of bridges based on total length intervals. Most structures are multi-span bridges having length values under 500 m.

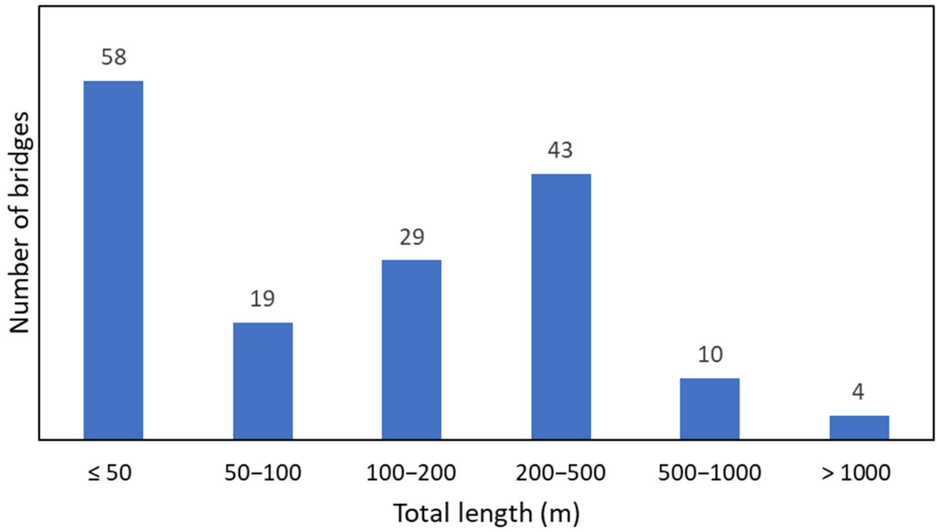


Figure 3. Classification of bridges by total length.

Figure 4 shows the total number of bridges distributed according to the different numbers of deck girders, which are defined based on the data reported in the database. It is observed that bridges with 3, 4, 5, and 6 girders have the highest number of occurrences. This condition justifies the choice of using these four values as the number of girders in the statistical analysis.

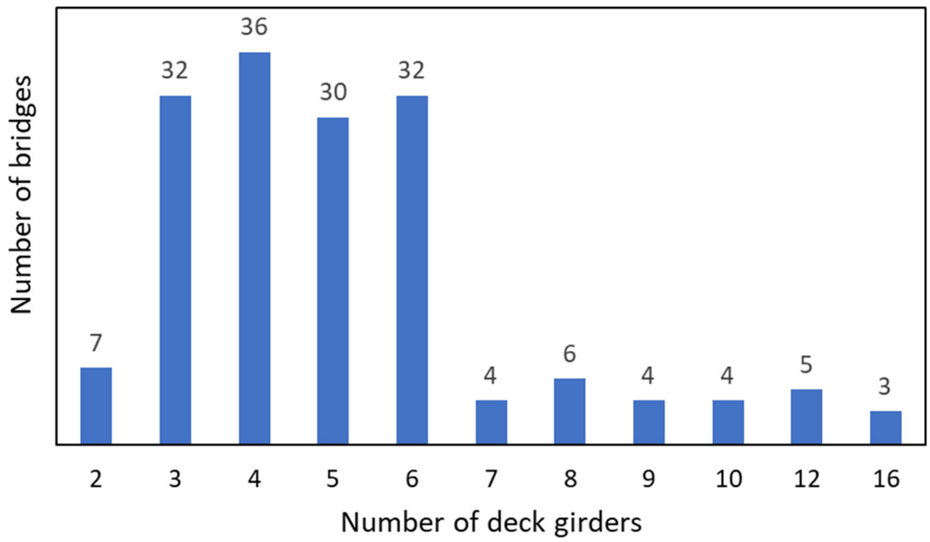


Figure 4. Classification of bridges by number of girders.

To carry out the Level 3 assessment according to the guidelines, a comparison based on traffic loads related to different regulations is conducted. In this study, the current code, namely the Ministerial Decree of 17 January 2018 "Technical Standards for Construction" [4], is compared with the following codes:

- Circular n°384 of 14 February 1962 [20], (C1962);
- Ministerial Decree of 2 August 1980 [21], (DM1980);
- Ministerial Decree of 4 May 1990 [22] (DM1990).

These are the regulations in force during the reference period, used for the design of the analysed bridges recorded in the database. Previous regulations for bridge load specifications [23–25] are not considered since the bridge stock considered was mainly built after 1962.

2.1. *Circular n°384 of 14 February 1962*

This code distinguishes between first-category and second-category bridges. In this analysis, the first category is considered to apply the maximum value of traffic loads and evaluate the most severe conditions.

The load schemes outlined in the code are as follows (Figure 5):

- Load Scheme 1: Indefinite column of 12-ton trucks;
- Load Scheme 2: Isolated 18-ton road roller;
- Load Scheme 3: Compacted crowd at a rate of 400 kg/m^2;
- Load Scheme 4: Indefinite train of military loads weighing 61.5 tons;
- Load Scheme 5: Indefinite train of military loads weighing 32 tons;
- Load Scheme 6: Isolated military load of 74.5 tons.

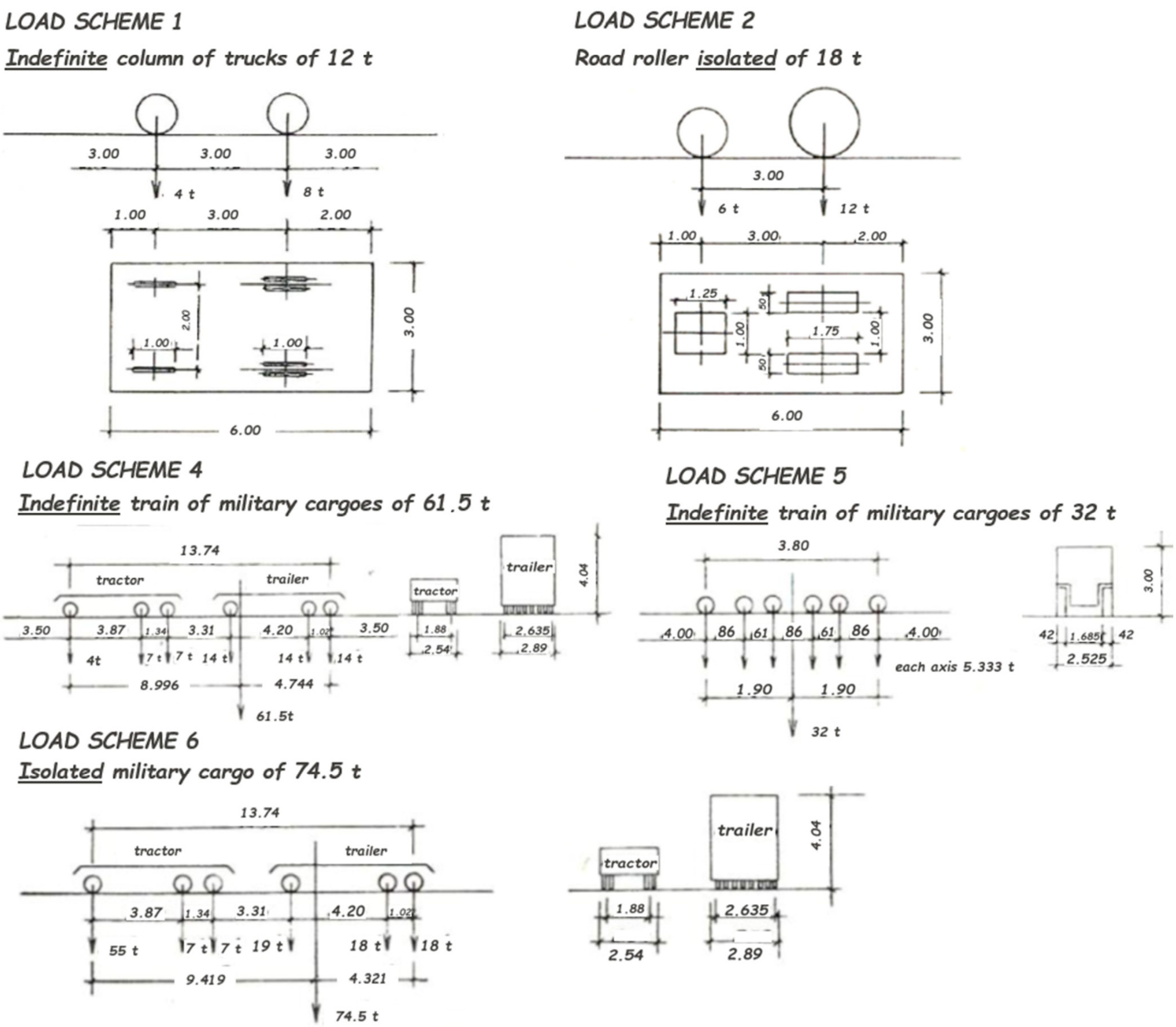


Figure 5. Military load schemes defined in Circular 1962.

For the design of bridges, this code prescribed the combination of a military load scheme (Schemes 4, 5 and 6) with a civilian load scheme (Scheme 1), considering the most severe conditions.

These loads are arranged to form load columns on lanes sized as follows:

- For military schemes (Schemes 4, 5 and 6), lane width of 3.50 m;
- For the civilian scheme (Scheme 1), lane width of 3 m.

Additionally, these loads are amplified using a dynamic amplification coefficient, which accounts for the dynamic nature of traffic loads above the deck. This coefficient is defined using the expression:

$$\phi = 1 + \frac{(100 - L)^2}{100(250 - L)} \tag{1}$$

where L is the span length.

2.2. Ministerial Decree of 2 August 1980

This regulation introduces the concept of conventional loads, which involve the presence of both concentrated and distributed loads. This configuration allows for the consideration of higher stress levels and, consequently, enables the evaluation of more realistic stress conditions in the bridge design.

The regulation distinguishes three bridge categories and the analysis refers to bridges in the first category, which is used for major road infrastructure. The specified live loads (expressed in ton and m) in the regulation are as follows:

- q_{1A}, related to a type A load column (distributed):
  - $2.89 + \frac{52}{L}$ for $L \leq 40$ m;
  - $4.35 - \frac{L}{250}$ for $40 \leq L \leq 400$ m;
  - 2.75 for $L > 400$ m.
- q_{1B}, related to a type B load column (distributed):
  - $0.40 + \frac{27}{L}$ for $L \leq 15$ m;
  - $2.23 - \frac{L}{500}$ for $15 \leq L \leq 400$ m;
  - 1.43 for $L > 400$ m.
- q_{1C} is a three-axle trailer (point loads) with a weight of 55 tons (Figure 6a);
- q_{1D} is a three-axle truck (point loads) weighing 31 tons (Figure 6b);
- q_{1E} is a load of 1 ton with a footprint area of 0.7×0.7 m^2;
- q_{1F} is the crowd load equal to 0.4 t/m^2.

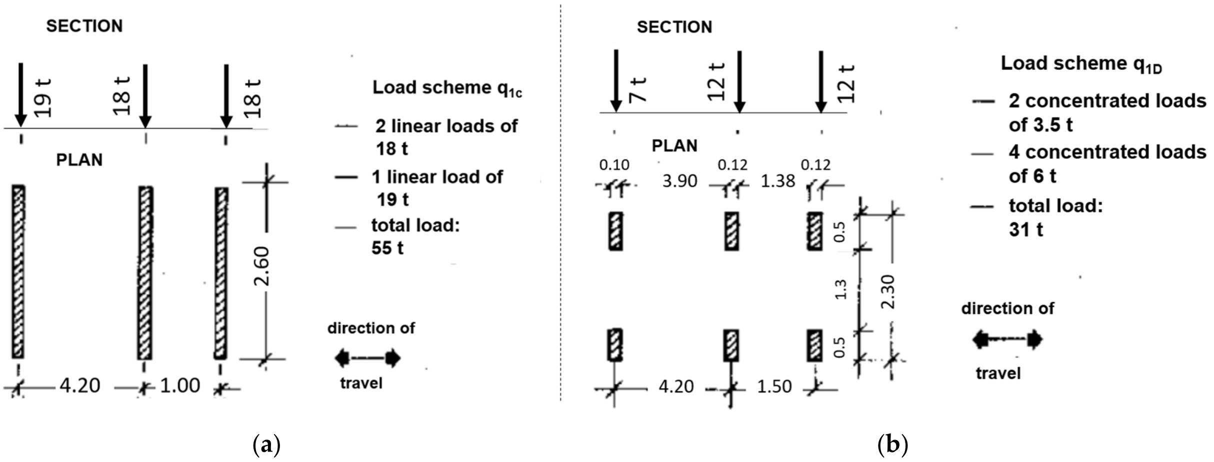


Figure 6. (**a**) Three-axle trailer with a total weight of 55 tons; (**b**) Three-axle truck with a total weight of 31 tons (dimension units in meters).

For the design of the first-category bridges, this code defines two load combinations. The First combination formed by:

- One load train q_{1A};
- One load train q_{1B};
- In the case of additional lanes, a reduced-intensity load q_{1B} with 30% intensity;
- In the presence of a sidewalk, a crowd load should also be considered.

The Second combination formed by:

- One load train q_{1C};
- One load train q_{1B};
- In the case of additional lanes, a reduced-intensity (30%) load q_{1B} is considered;
- In the presence of a sidewalk, a crowd load should also be considered.

These loads should be arranged on appropriately sized lanes, assuming a conventional clearance width of 3.50 m. The minimum number of lanes is two. Only for road platform widths lower than 5 m, the minimum number of lanes can be assumed as one.

Furthermore, the loads are amplified using the dynamic amplification coefficient, which can be defined using the expression:

$$\phi = 1.4 - 0.002\left(\frac{g}{q} + 1\right)L \quad (2)$$

From an analysis of the above expression, the coefficient Φ depends on the parameter g/q, which is the ratio between the dead load and the design traffic load.

For the subsequent parametric analysis, generated by the traffic loads specified in the 1980 Ministerial Decree, the dynamic amplification coefficient Φ is calculated by precisely defining the g/q parameter.

In particular, the dead load g is calculated concerning all the elements characterizing the cross-section of the bridge deck (defined in the deck layout section in the following chapter), namely:

- Thickness of reinforced concrete slab according to Table 1;
- 25 cm deep reinforced concrete curb;
- 10 cm thick road pavement;
- Metallic guard rail;
- Longitudinal reinforced concrete girders.

Table 1. Slab thickness assumptions.

Number of Girders	Slab Thickness (cm)
3	30
4	28
5	25
6	25

For the latter, since the geometric dimensions of the cross-section are not known, the dead load g is defined using pre-dimensioning formulas and utilizing design documents of existing bridges.

The total dead load g is compared to each traffic load, thus defining the dynamic amplification coefficient, which is applied to each load component in the two load combinations mentioned above.

2.3. Ministerial Decree of 4 May 1990

In this standard, the approach defined in the previous standard is reaffirmed, which involves the presence of concentrated and distributed loads. The code distinguishes between three categories of bridges, and the analysis refers to bridges of the first category.

The specified moving loads in the standard are (Figure 7):

- q_{1A}: conventional vehicle load of 60 tons with 3 axles;
- q_{1B}: distributed load of 3 t/m distributed, for the calculation of main structures, along the axis of a traffic lane;
- q_{1C}: isolated load of 10 tons with a square footprint of 0.3 m side length;
- q_{1D}: isolated load of 1 ton with a square footprint of 0.7 m side length;
- q_{1E}: uniformly distributed crowd load of 0.4 t/m over the surface.

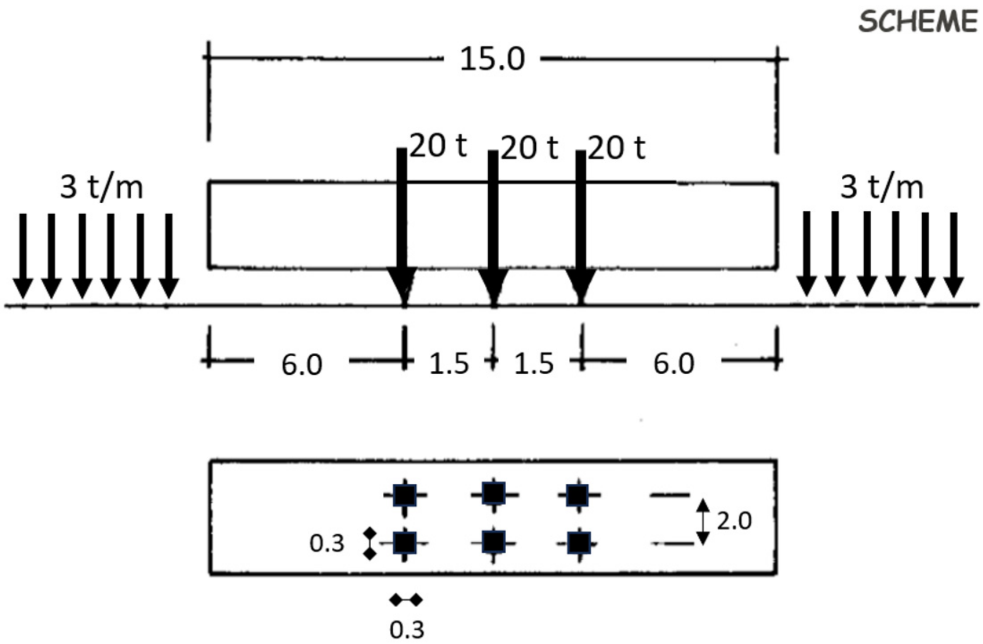


Figure 7. 60 tons vehicle with 3 axles plus distributed load.

For the design of bridges in the first category, the load combinations to be defined are as follows:

- A load combination consisting only of q_{1A}, which is the conventional vehicle load. Outside the footprint of the conventional vehicle, the load q_{1B}, with an intensity of 3 t/m, must be placed;
- A second load combination consisting of both q_{1A} and q_{1B}, reducing their intensities by 50%;
- A third or additional load combinations consist of q_{1A} and q_{1B}, with intensities reduced by 35%;
- In the presence of sidewalks, the crowd load with an intensity of 0.4 t/m is also considered.

These loads are placed on conventional traffic lanes, dimensioned according to the standard. The conventional lane width is 3.50 m, and the minimum number of lanes is defined as two. Only for road platforms narrower than 5.50 m, the minimum number of lanes can be defined as one. Furthermore, to account for the dynamic nature of traffic loads, they are amplified using the dynamic amplification coefficient, which can be defined using the formula:

$$\phi = 1.4 - \frac{(L - 10)}{150} \tag{3}$$

2.4. Ministerial Decree of 17 January 2018

This is the current standard used for bridge design in Italy, providing guidelines for the design of road bridges. The standard refers to two types of bridges: the first category bridges and pedestrian bridges. In this analysis, we focus on the first-category bridges.

There are a total of six load patterns defined in the standard and the load pattern used to calculate the stress values is Load Pattern 1 (see Figure 8), which is used for global safety checks during the design process.

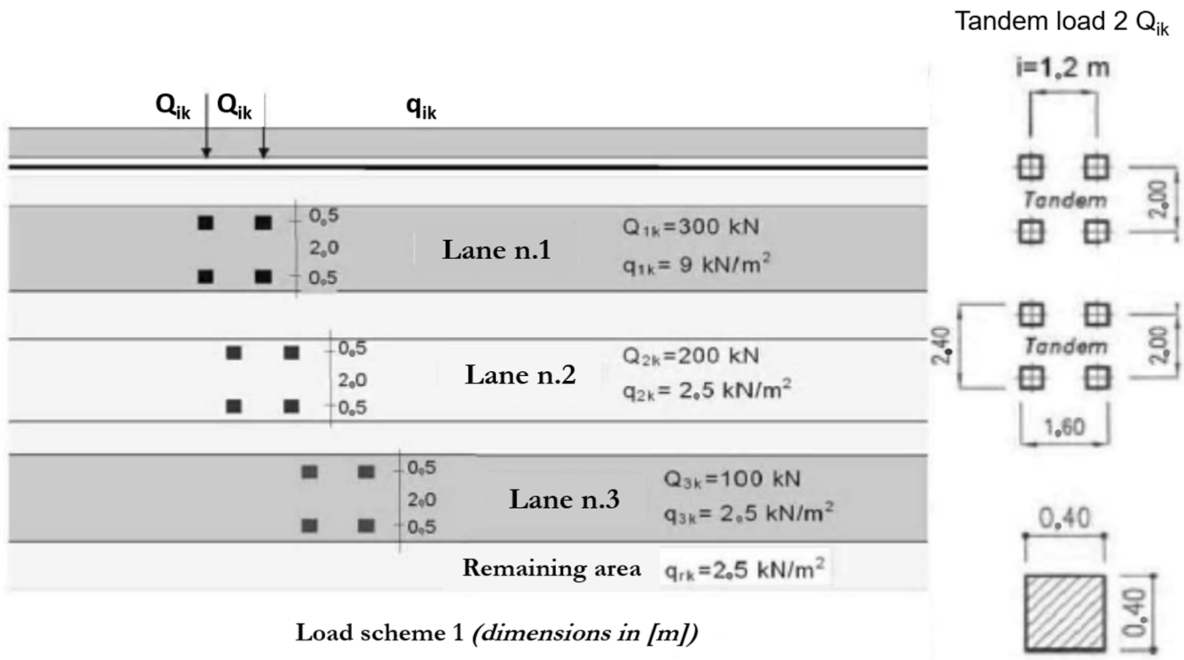


Figure 8. Load schemes according to DM2018.

The current standard does not require the calculation of the dynamic amplification that is already incorporated by the loading schemes.

These loads are applied on conventional lanes, based on the roadway width, and the maximum number of lanes can be defined as three (Figure 9):

Carriageway width "w"	Number of conventional lanes	Width of a conventional lane "m"	Width of the remaining area "m"
$w < 5.40$ m	$n_l = 1$	3.00	$(w - 3.00)$
$5.40 \le w < 6.0$ m	$n_l = 2$	$w/2$	0
6.0 m $\le w$	$n_l = \mathrm{Int}(w/3)$	3.00	$w - (3.00 \times n_l)$

Figure 9. Number and size of lanes according to DM2018.

3. Definition of the Girder Deck Schemes

In order to define statistically representative girder deck types, the attention is focused on three bridge properties:

- Deck width;
- Number of longitudinal girders;
- Spacing between girders.

These parameters are defined through a statistical study rather than deterministically, meaning that a unique value is not assumed for each of them to define the geometry of the bridge deck section.

Therefore, these three parameters are treated as random variables. However, since they are at least partially dependent on each other, the statistical analysis is simplified by defining the number of girders as an independent variable while the deck width and, consequently, the spacing between girders are dependent variables.

The deck width is considered a reference for developing the statistical analysis. The implemented statistical model assumes that the statistical population, i.e., the data related to the deck width, follows a known probability distribution. In this case, it is assumed to be a Gaussian probability distribution, also known as a normal distribution.

Analysing the database, the values of the two parameters on which the Gaussian probability distribution depends, namely the mean and the standard deviation, can be computed (Table 2).

Table 2. Parameters for probability distributions.

Number of Deck Girders	Number of Bridges	Average Deck Width (m)	Standard Deviation of Deck Width (m)	Average Size of Deck Cantilever (m)
3	32	9.55	1.74	0.38
4	36	9.39	0.75	0.46
5	30	11.18	1.45	0.44
6	32	10.99	1.41	0.50

Once the two parameters are known, the shape of the probability density function is defined and depicted graphically (Figure 10), putting the deck width along the x-axis. Overall, there are four representations of the probability distribution, corresponding to the four values associated with the data regarding the number of girders n.

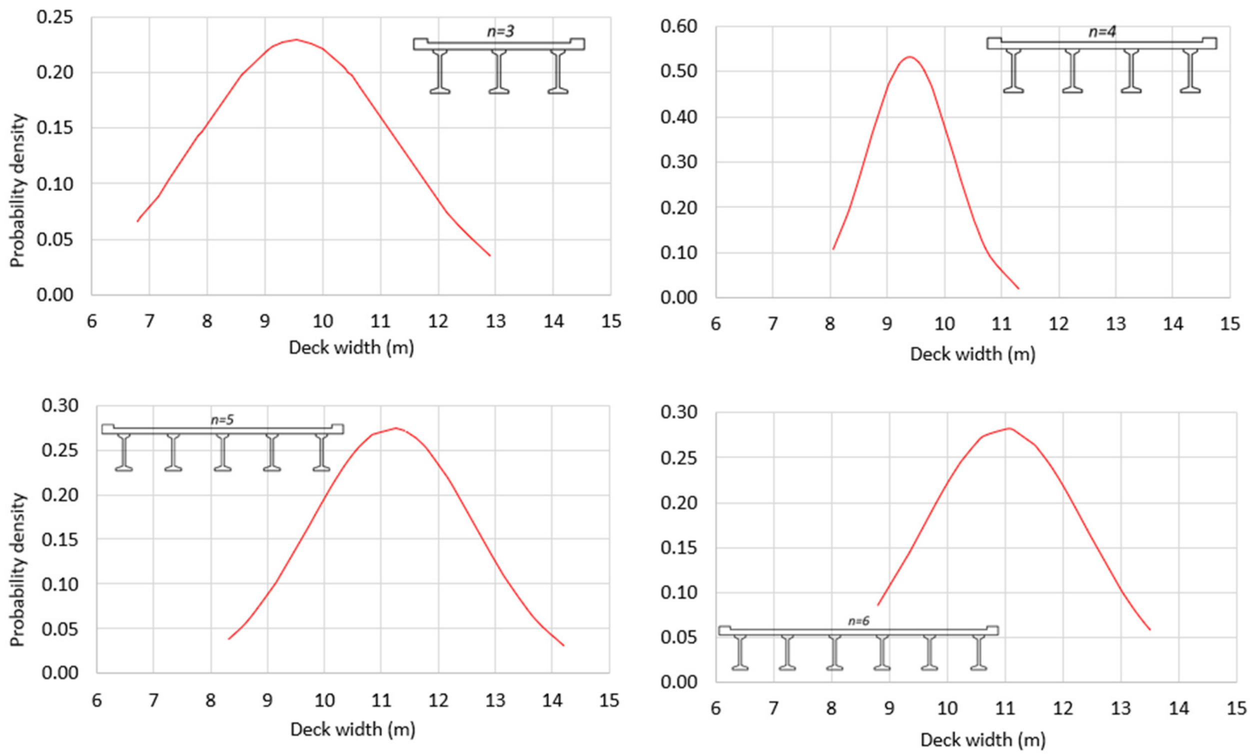


Figure 10. Deck width probability distributions.

Latin Hypercube Sampling (LHS)

In order to sample from the probability distributions deck width values representative of the bridge database, the latter are extracted using the Latin Hypercube Sampling (LHS) technique. Being a stratified sampling, it allows for a statistically representative description of the stock even with a limited number of samples.

The steps carried out for sampling are as follows:

(1) Definition of the cumulative probability function;

(2) Stratification into five equal intervals on the y-axis, i.e., for the values 0–0.2–0.4–0.6–0.8–1;

(3) Definition of the mean line corresponding to the centroid of each interval, i.e., for the values 0.1–0.3–0.5–0.7–0.9 of the cumulative function, and determining their intersection with the cumulative density function and, then, with the horizontal axis to determine the deck width values (Figure 11).

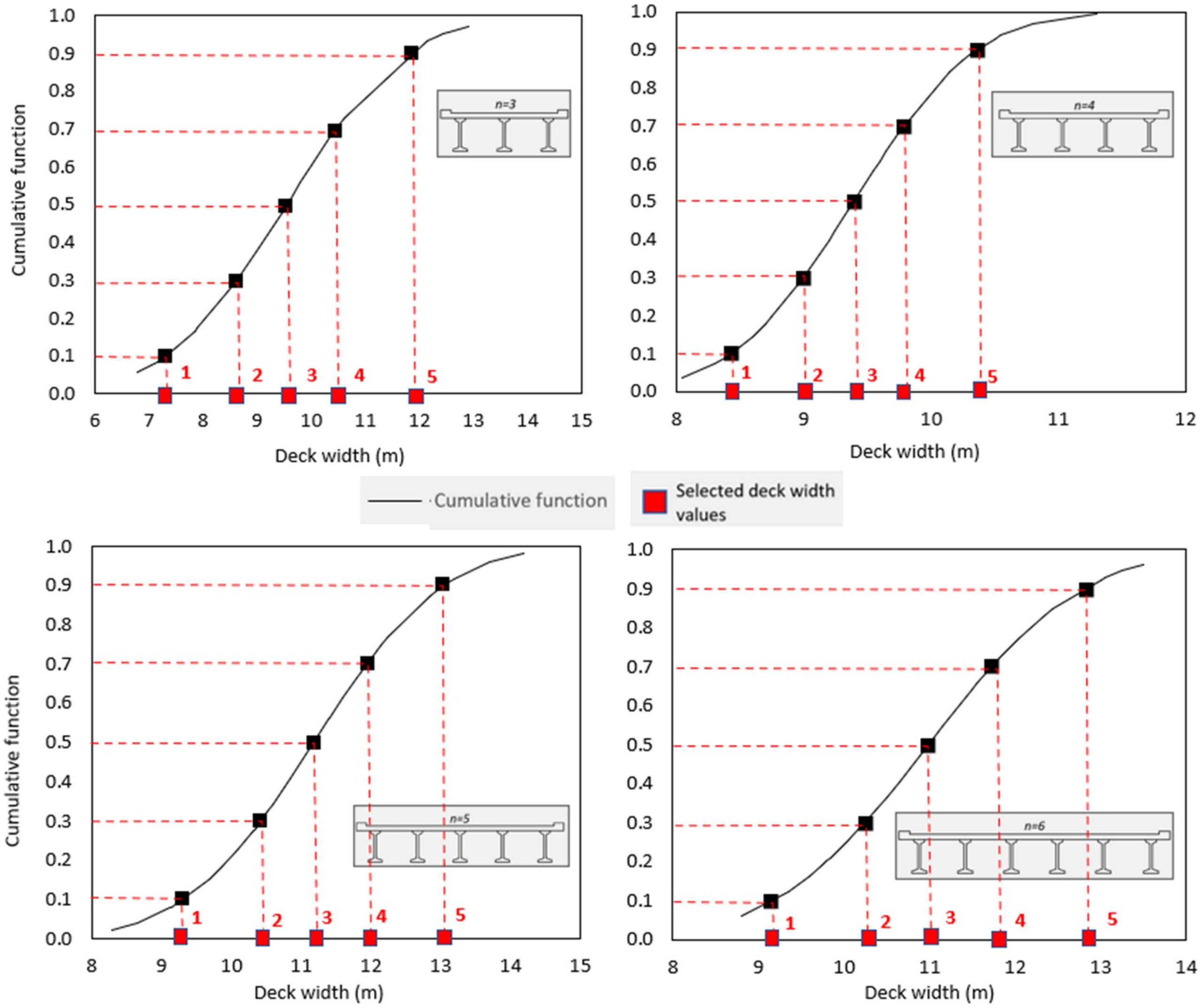
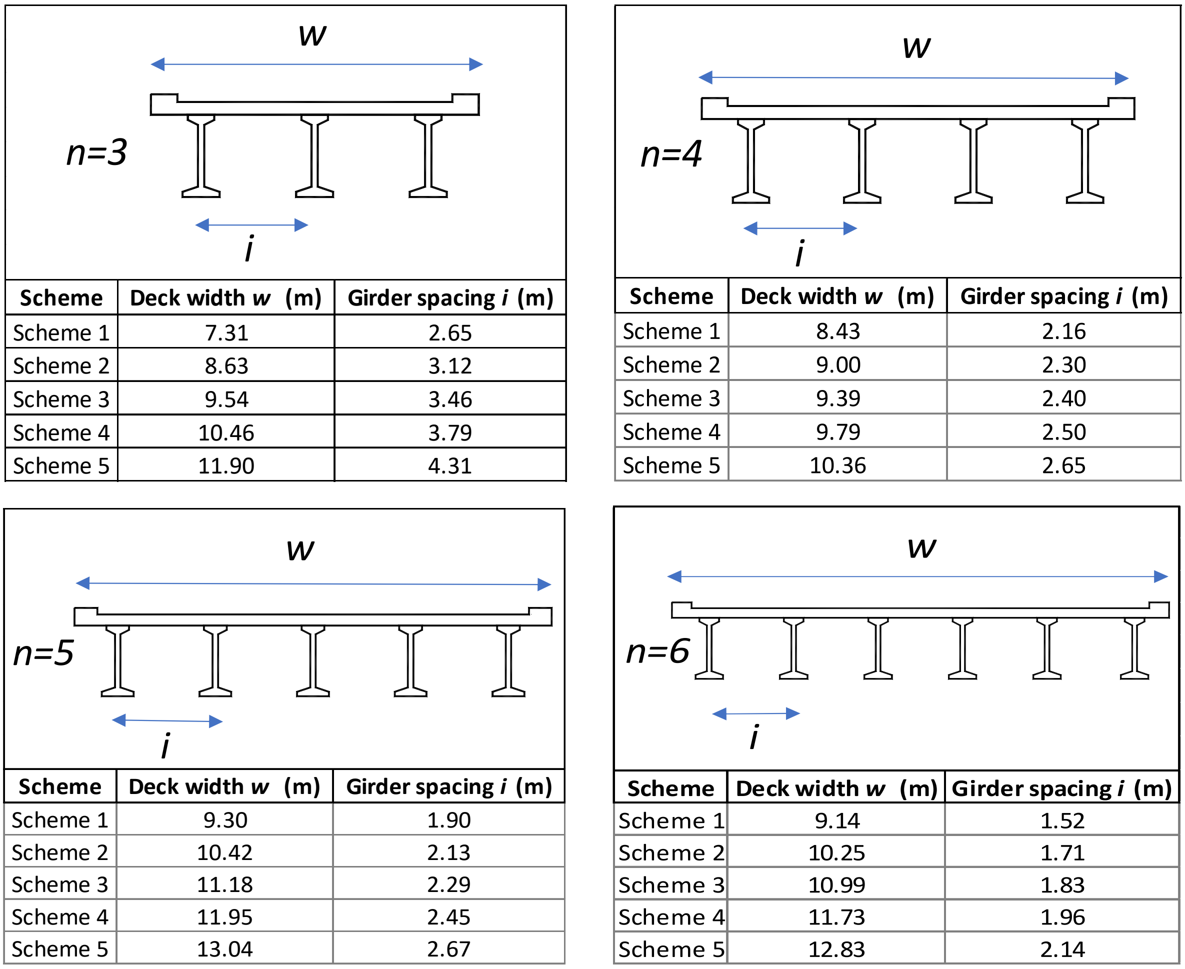


Figure 11. Latin Hypercube Sampling for the different number of girders.

At the end of the statistical analysis, five girder deck schemes (red squares in Figure 11) are obtained for each of the four values of n, which are subsequently analysed considering four values of the span length L (20, 30, 40, 50 m). Therefore, a total of $5 \times 4 \times 4 = 80$ configurations is obtained (Figure 12).

It is worth noting that deck width values for $n = 5$ are slightly larger than those for $n = 6$. This means that decks with 5 and 6 girders can be considered almost equivalent.



Scheme	Deck width *w* (m)	Girder spacing *i* (m)
Scheme 1	7.31	2.65
Scheme 2	8.63	3.12
Scheme 3	9.54	3.46
Scheme 4	10.46	3.79
Scheme 5	11.90	4.31

Scheme	Deck width *w* (m)	Girder spacing *i* (m)
Scheme 1	8.43	2.16
Scheme 2	9.00	2.30
Scheme 3	9.39	2.40
Scheme 4	9.79	2.50
Scheme 5	10.36	2.65

Scheme	Deck width *w* (m)	Girder spacing *i* (m)
Scheme 1	9.30	1.90
Scheme 2	10.42	2.13
Scheme 3	11.18	2.29
Scheme 4	11.95	2.45
Scheme 5	13.04	2.67

Scheme	Deck width *w* (m)	Girder spacing *i* (m)
Scheme 1	9.14	1.52
Scheme 2	10.25	1.71
Scheme 3	10.99	1.83
Scheme 4	11.73	1.96
Scheme 5	12.83	2.14

Figure 12. Deck schemes for the n values.

4. Application of Load Schemes and Stress Analysis

On the girder deck configurations, after designing the conventional lanes according to code indications (see Sections 2.1–2.4), the traffic load patterns are applied, referring to the first category bridges for each norm.

Then, the procedure of traffic load transverse distribution is performed, using the Courbon Engesser method [16]. By calculating the transverse distribution coefficient r_i (4) the portion of the traffic load P transmitted to the longitudinal girders, constituting the girder deck configurations, is determined (Figure 13).

$$r_i = \frac{1}{n} \pm \frac{ed_i}{\sum d_i^2} \tag{4}$$

where:

- e represents the distance between the centroid of the cross-sectional area of the girder deck configuration and the point of application of the resulting load;
- n represents the number of deck girders, all having the same cross-section and therefore identical stiffness k;
- d_i represents the distance of each individual girder from the centroid G.

In Figure 13, a general girder deck configuration is shown, indicating the parameters used to perform the transverse load distribution procedure.

In the calculation of the stress values, the focus is on the edge longitudinal girders of each girder deck configuration because they are the most loaded and therefore subject to higher stresses.

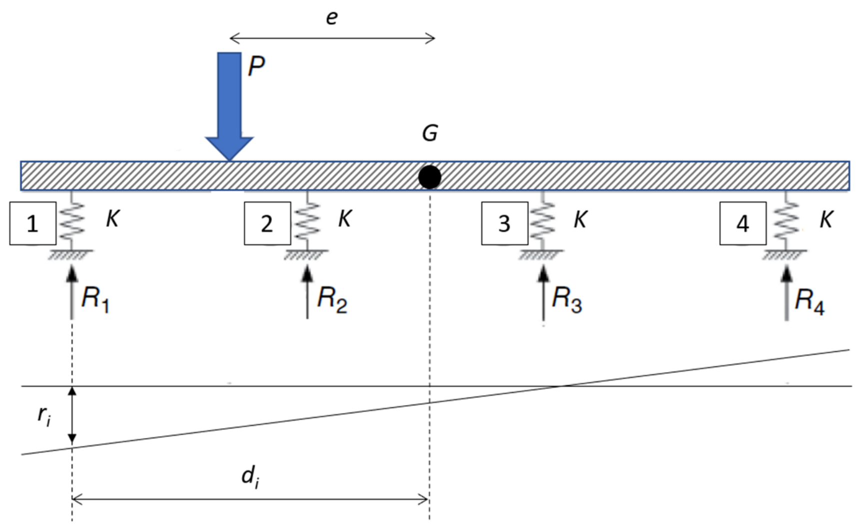


Figure 13. Scheme for the evaluation of transverse distribution coefficient r_i.

The bending and shear stress values are calculated using influence lines. Starting from the statically determined diagram of a simply supported girder and applying the loads on the edge girder of each girder deck configuration, the influence line that maximizes the bending stress and the one that maximizes the shear stress are used.

A particularity related to the calculation of stress characteristics induced by the load patterns mentioned in C1962 (see Section 2.1) is highlighted. Military load patterns 4 and 5 and civilian load pattern 1 are referred to as indefinite loads, indicating that there is an endless number of vehicles on the bridge deck covering the whole length. Consequently, to calculate the stress characteristics generated by these three load patterns, the research work has involved generating subroutines using the Visual Basic for Applications (VBA) editor in Microsoft Excel.

Through this procedure, it is assumed that the load moves along the deck, taking into account that the number of vehicles gradually increases. By setting the ordinates of the influence lines for each load acting on the deck, the magnitude of both bending and shear stresses are obtained.

In total, 526 subroutines are generated, considering all girder deck configurations with different numbers of girders and span lengths.

It is worth noting that for DM1980 and DM1990 the maximum bending stress values are found for load positions near the mid-span length, while for C1962, due to the more complex arrangement of vehicles, the most severe position is found through the above-mentioned routines as the span length changes.

Once the stress values are obtained for the outdated standards (C1962 [20], DM1980 [21], and DM1990 [22]), they are compared with those calculated according to the current standard (DM2018 [4]). A preliminary Level 3 assessment is carried out, as indicated in the guidelines, by defining dimensionless performance indices *I*, which are derived from the aforementioned ratio, accounting for moment stress (*M*) and shear (*T*), through Expressions (5) and (6):

$$I_M = \frac{M_{old\ code}}{M_{2018}} \tag{5}$$

$$I_T = \frac{T_{old\ code}}{T_{2018}} \tag{6}$$

I_M and I_T values larger than 1.0 mean that design stress values are larger than those used for the assessment according to the current code. On the other hand, values lower than 1.0 are representative of design stress values lower than those related to the current code, identifying a design capacity gap. This gap could be even larger in cases where the self-weight of the bridge was not properly evaluated during the design phase [26].

While I_M is related to the adequacy of girders, whose design is dominated by flexural stress, I_T gives indications not only on the shear design resistance of girders but also regarding the design vertical action of bearing devices. In fact, the maximum shear in simply supported beams corresponds to the vertical reaction of constraining devices.

It is worth noting that I_M and I_T may be useful to prioritize further actions on bridges (Level 4 detailed assessment or interventions). In fact, the class of attention results obtained at Level 2 assumes only five values (low, medium-low, medium, medium-high and high), therefore, is not suitable for creating risk rankings in the case of large-scale bridge stocks. Indeed, over a large bridge inventory, many bridges may have the same class of attention [10] and no decision can be made on which ones should have priority. Using I_M eventually coupled with I_T values can provide a numerical result to rank the bridges with the same class of attention.

5. Results and Discussion

The performance gap of bridges designed according to the regulations in force at the time of their design is highlighted when applying the traffic load models specified in the current regulations used for designing new structures. The dimensionless performance indices are represented through graphs, where the span length of the deck is plotted on the *x*-axis and I_M and I_T values on the *y*-axis (Figures 14–19).

Analysing the graphs, it is evident that bridges designed in the past according to the old regulations exhibit a performance gap when subjected to the traffic load models specified in the current regulations. Furthermore, it is observed that this gap tends to decrease as the regulations, in terms of implementation timeframe, approach the current ones.

Performance indices I_M for the C1962 code are in the range of 0.6–0.9. The gap (1–I_M) is larger for lower span length values and for schemes with smaller deck width (e.g., scheme 1). This is due to the fact that the current code considers a maximum of three lanes while C1962 considers a number of lanes dependent on the deck width. A similar trend is observed for I_T that achieves slightly higher values (almost equal to 1.0) for higher span length values.

The DM1980 code shows I_M indices around 0.80 for the extreme values of span length L = 20 and L = 50 m, with lower values for the intermediate spans. Similar patterns are obtained for I_T values, which are even lower, in the range of 0.7–0.8.

Finally, the DM1990 code yields I_M values almost insensitive to the span length and included in the range of 0.80–0.85. A similar constant distribution is observed for I_T, which is slightly higher, in the range of 0.85–0.90.

Thanks to the introduction of distributed loads in loading schemes, I_M and I_T distributions of the DM1980 and DM1990 codes are not very sensitive to changes in span length. On the contrary, C1962 results are more sensitive to increases in the span length values due to the presence of military trains, which are more and more numerous when the span length is larger.

In order to easily compare the results, a performance coefficient β_m has been set. It is also devoted to describing the impact of the transverse load distribution procedure on the results of the parametric analysis and, consequently, on the outcome of the preliminary assessments. β_m is computed referring to bending stress, for each number of girders and each code, using the formula:

$$\beta_m = \frac{\sum_{s=1}^{5}\left(I_{M(L=30)}\right) + \sum_{s=1}^{5}\left(I_{M(L=40)}\right)}{10}. \quad (7)$$

Indeed, β_m is the average of the 10 performance index values I_M computed considering the five deck schemes (s = 1 to 5) for the span length values equal to 30 and 40 m. The choice of these two span length values depends on the observation that most simply supported bridges in Italy have a mean span length of around 33 m [19], which is in the range of 30–40 m.

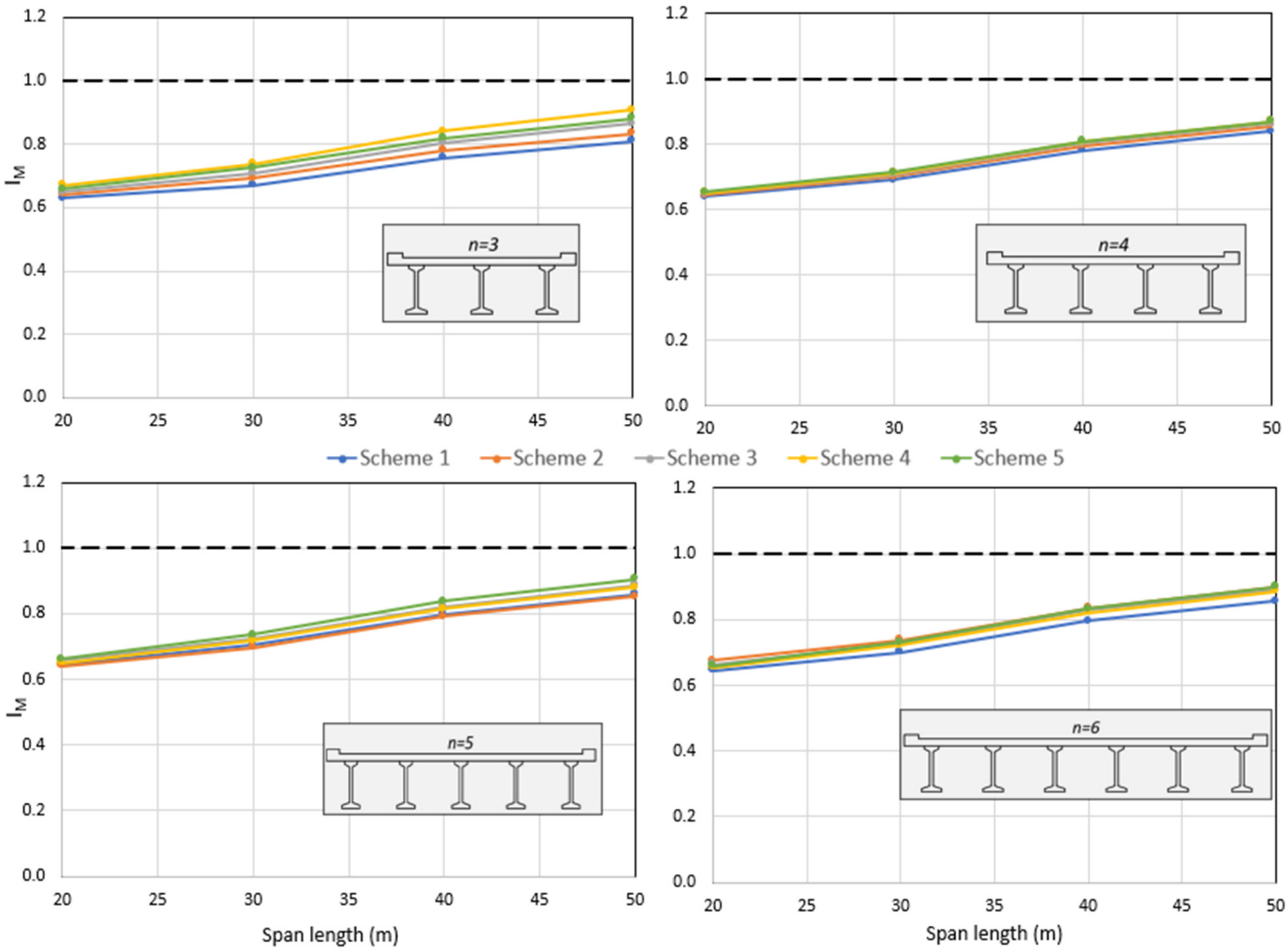


Figure 14. Performance indices I_M for Circular 1962.

The performance coefficients are calculated only for the bending stress in order to compare the results of the present study with those of similar research that did not consider the transverse load distribution [15]. This latter, in fact, considered only the bending stress values. The β_m values are shown in Figure 20 for both the present study (n = 3, 4, 5, 6) and the one reported in [15].

Figure 20 first highlights that the performance coefficients computed without transverse load distribution [15] assume higher values compared to those calculated using the performance indices I_M defined in the analysis carried out in this study, implementing the transverse load distribution procedure. This result implies that the performance gap, which is evident in the preliminary assessment according to the guidelines, is higher when the evaluation is performed by applying the transverse load distribution procedure.

For C1962, β_m is averagely equal to 0.76 when accounting for the transverse load distribution, and 0.92 without it. For the DM1980 code, the comparison gives 0.72 vs. 0.95. Lastly, the DM1990 code presents values averagely equal to 0.79 vs. 0.89. Therefore, neglecting the transverse load distribution can lead to largely underestimating the flexural strength gap of about 15% and 23% for the C1962 and DM1980 codes, respectively, and of about 10% for the 1990 code.

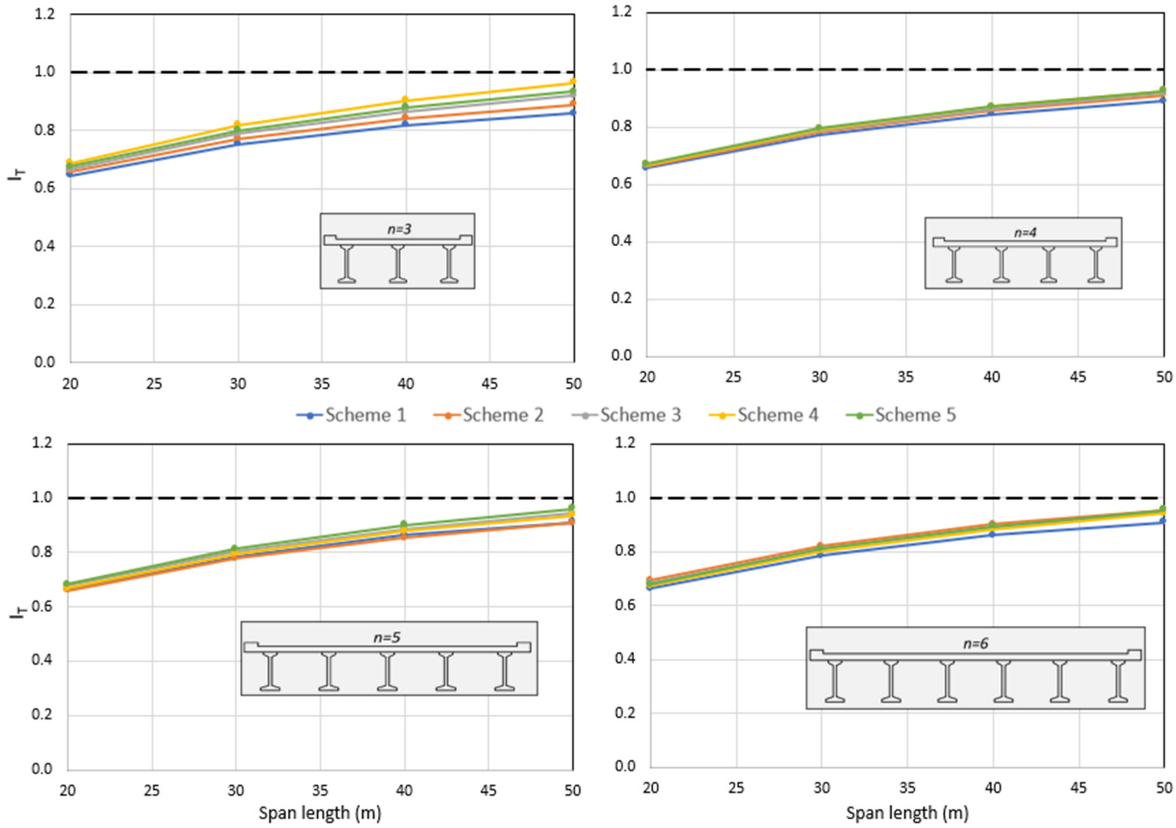


Figure 15. Performance indices I_T for Circular 1962.

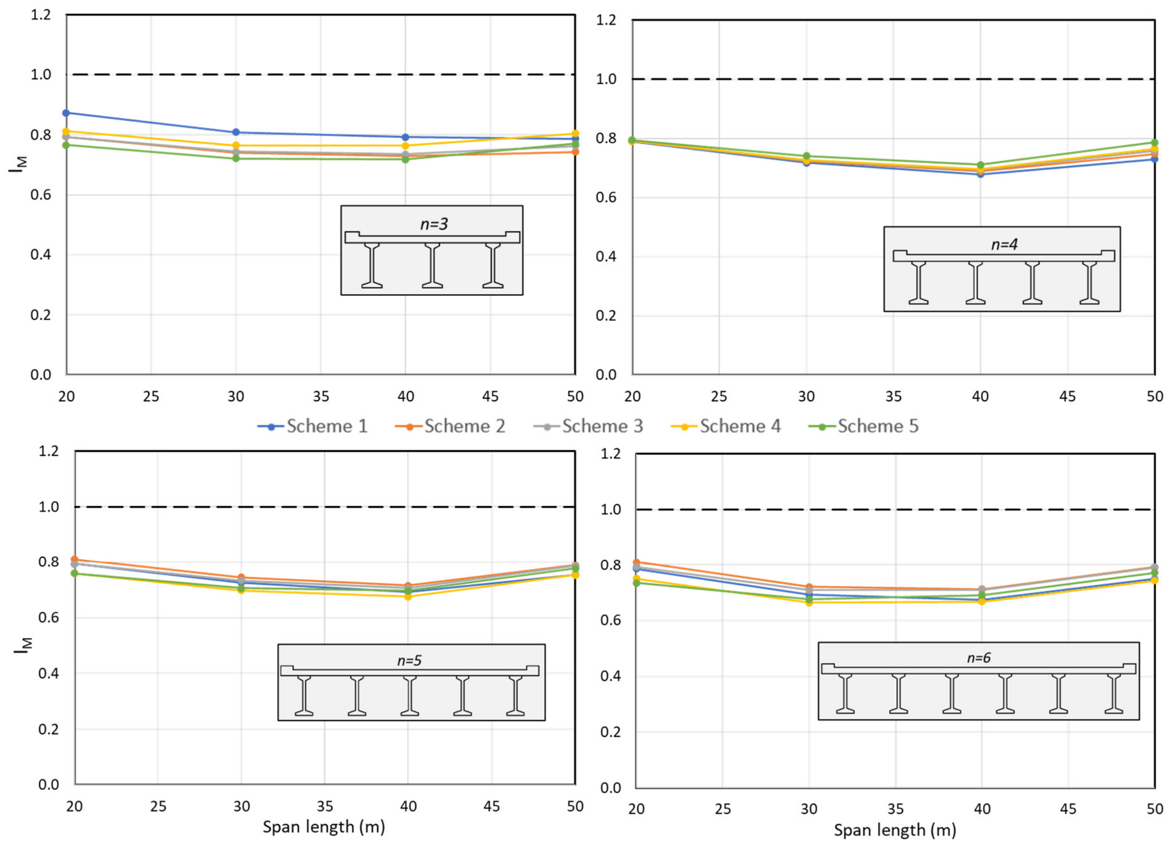


Figure 16. Performance indices I_M for DM1980.

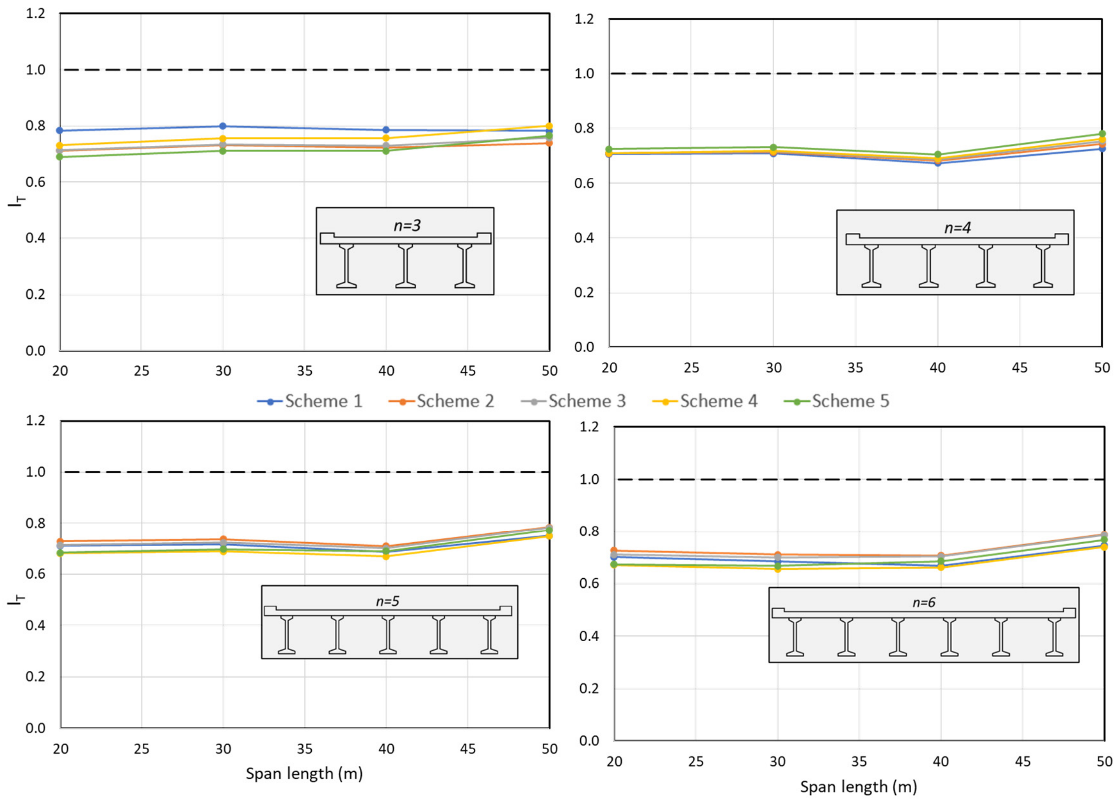


Figure 17. Performance indices I_T for DM1980.

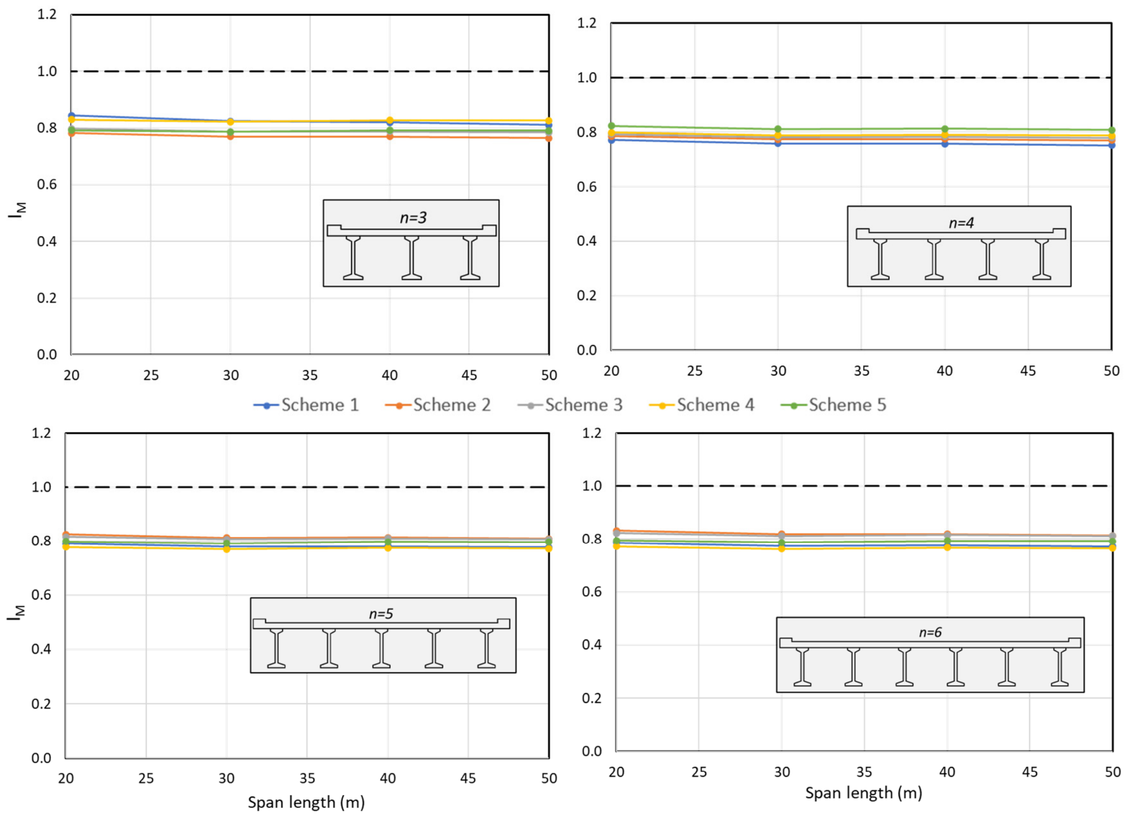


Figure 18. Performance indices I_M for DM1990.

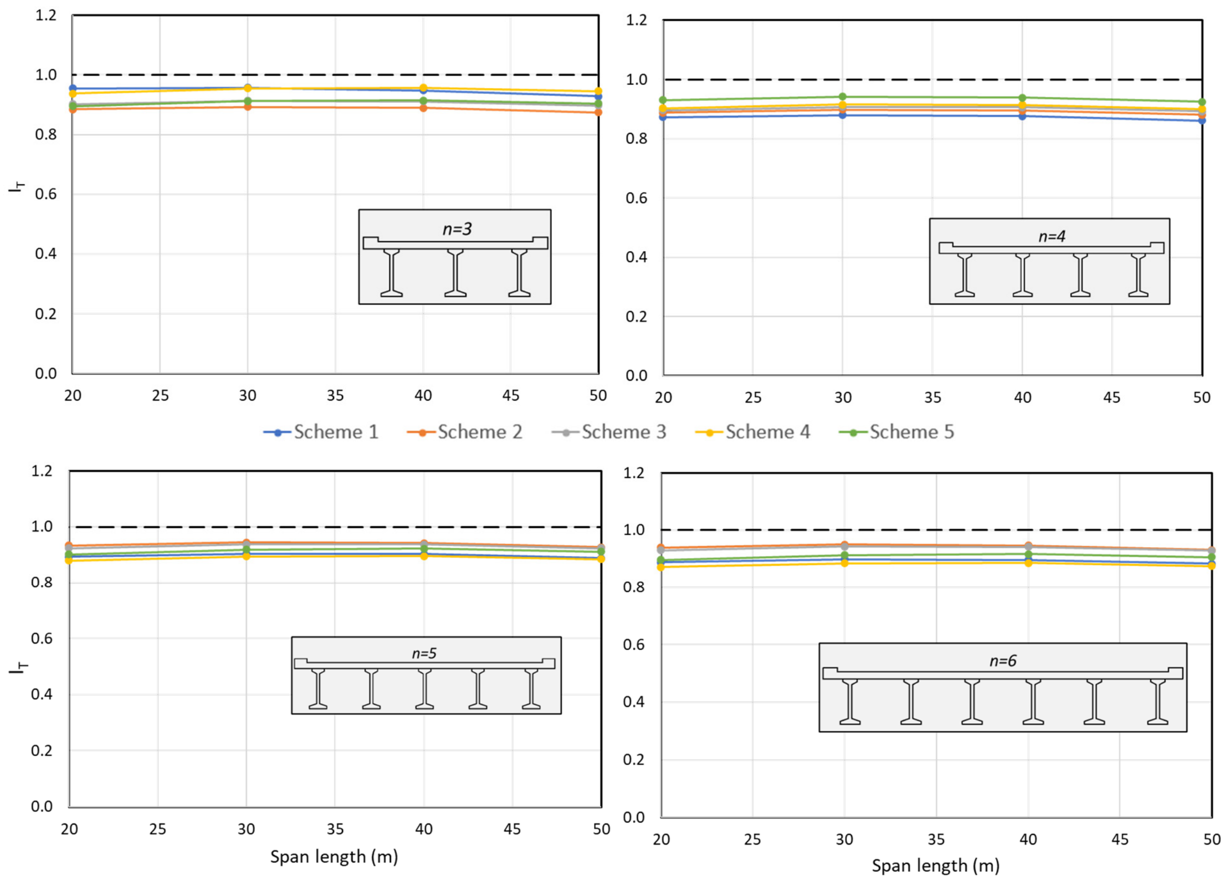


Figure 19. Performance indices I_T for DM1990.

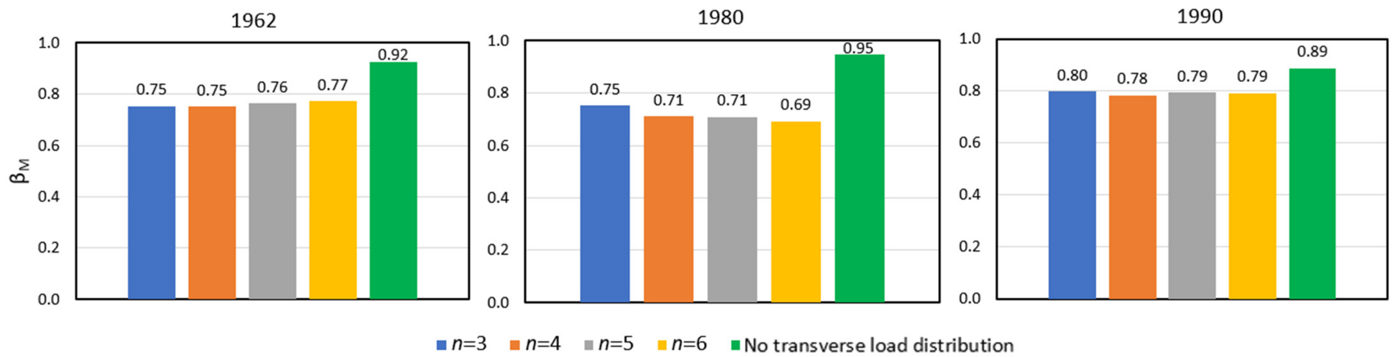


Figure 20. Performance indices β_m.

6. Conclusions

This paper presents the results of a parametric analysis related to the preliminary assessment procedure prescribed at Level 3 of the Italian guidelines for existing bridges. A database of state roads simply supported PC bridges located in the Basilicata region (Italy) has been collected and used to extract representative girder deck cross sections to be subjected to preliminary assessment, comparing design stress values derived from older codes to those related to the application of the current code, DM2018.

The main results obtained from the analysis are as follows:

- The performance indices obtained from the normative comparison are all less than one for both bending and shear stress values;

- The performance indices according to the oldest code (C1962) are the most variable with respect to the span length due to the absence of distributed loads that were introduced only in the more recent codes (DM1980 and DM1990);
- Wider cross sections (schemes 4 and 5) generally lead to higher performance indices and, then, to lower strength gaps;
- Significant overestimation of the performance indices can be obtained when neglecting the transverse load distribution.

The obtained values of the performance indices can represent a useful reference for fast Level 3 assessment regarding state road bridges with geometric properties similar to those considered in this study. It is worth noting the flexural performance index is helpful to estimate the main girders' strength design gap, while the shear index is also representative of vertical force values used in the bearings' design.

Moreover, preliminary evaluations can be used to prioritize further actions on bridges (accurate assessment or interventions) since the class of attention, which assumes only five values, is not suitable to create risk rankings for large-scale bridge stocks. In fact, over a large bridge inventory, many bridges may have the same class of attention and no decision can be made on which ones should have priority.

Author Contributions: Conceptualization, G.S., A.M., P.A. and V.P.; methodology, G.S. and P.A.; software, P.A.; validation, G.S., A.M. and P.A.; formal analysis, G.S., P.A., V.P., A.M. and G.V.; investigation, G.S., P.A. and V.P.; resources, G.S. and A.M.; data curation, G.S. and P.A.; writing—original draft preparation, G.S., P.A. and V.P.; writing—review and editing, G.S., P.A. and G.V.; visualization, G.S. and P.A.; supervision, G.S. and A.M.; project administration, G.S., A.M. and G.V.; funding acquisition, G.S., A.M. and G.V. All authors have read and agreed to the published version of the manuscript.

Funding: This research is funded by CSLLPP (High Council of Public Works as part of the Italian Ministry of Infrastructure and Transportation) in the framework of the agreement with the ReLUIS consortium for the research project "Accordo attuativo del DM 578 del 2020 a 204 del 2022" WP4, Task 4.2.

Data Availability Statement: No data is available.

Acknowledgments: The first author wishes to thank Eng. Antonio Musano for his contribution to this study through several fruitful discussions and his B.S. thesis on the topic.

Conflicts of Interest: The authors declare no conflict of interest.

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